Steady-State Strength Analysis of Lower San Fernando Dam Slide[a]

Discussion by Robert W. Day,[5] Fellow, ASCE

The authors present a very interesting and well-prepared paper on the failure of the Lower San Fernando Dam, as a result of the 1971 Richter Magnitude 6.6 San Fernando earthquake, in Southern California. The authors indicate that based on data from seismoscopes, the slide started 40 s after the earthquake shaking had stopped, and thus it was not the inertia forces induced by the earthquake that caused failure, but rather the loss of strength due to liquefaction. The authors agreed to a consensus opinion that the actual field undrained steady-state stress (back-calculated from the actual slide) was in the range of 400–500 psf, which is lower than the average strength results from laboratory tests (610–810 psf).

It seems unusual that the dam did not fall until 40 s after the earthquake shaking had stopped. Could it be possible that liquefaction caused blocks 9–11 (Fig. 1) located at the toe of the slope to fail during the earthquake? This would then reduce toe support for the rest of the dam, and then perhaps blocks 5–8 failed. Finally at the crest of the dam, due to a lack of substantial toe support, the final blocks 1–4 slumped downward 40 s after the earthquake had stopped shaking. In this scenario, because the inertial earthquake forces contributed to the failure, a higher value of the actual field undrained steady-state stress (perhaps even close to the strength from laboratory tests) would be required. Do the authors think it is possible for this progressive failure to have occurred at the Lower San Fernando Dam?

Discussion by H. John Hovland,[6] Member, ASCE

This paper assesses the value of the undrained steady-state strength (SSS) concept presented by Poulos et al. (1985) in explaining the 1971 failure of Lower San Fernando Dam. The application of the SSS concept successfully shows that the upstream side of the dam was susceptible to failure. However, based on carefully conducted laboratory and field testing, and analysis, the results indicate that the strength mobilized in the field during the failure was less than half of the corrected laboratory strength estimates. Thus, it would appear that the important SSS concept does not fully explain the failure of Lower San Fernando Dam.

According to Poulos et al. (1985), "The steady state of deformation for any mass of particles is that state in which the mass is continuously deforming at constant volume, constant normal effective stress, constant shear stress, and constant rate of shear strain." Are these restrictions satisfied in the field, as they can be in the laboratory?

The writer wishes to draw attention to certain aspects of the failure, as depicted in Fig. 1(*a*).

---

[a]March, 1992, Vol. 118, No. 3, by Gonzalo Castro, Raymond B. Seed, Thomas O. Keller, and H. Bolton Seed (Paper 147).
[5]Chief Engr., American Geotech., 5764 Pacific Ctr. Blvd., Suite 112, San Diego, CA 92121.
[6]Consulting Geotech. Engr., 781 Alvarado Road, Berkeley, CA 94705.

1. Block 10 totally separated from block 7. That is, the toe (especially blocks 10 and 11) moved faster and away from the upslope blocks. At this end stage of the failure, the upper blocks 1–6 were not pushing the lower blocks. Instead, it appears that the upper blocks failed, or had failed, in shear as the liquefied toe flowed away. The rate of shear strain could not have been constant throughout.

2. Liquefied soil is shown approximately up to the rolled fill-ground shale hydraulic fill interface. As shown, liquefied soil flowed up into these cracks, probably against the reservoir pressure. Such cracks, whether vertical or horizontal, as they were being filled with liquefied soil, are analogous to a grout slurry being pumped into the ground; the driving force comes from the pressure, particles are in suspension, and the normal effective stress, as well as the effective major and minor principal stresses, are zero. This suggests a possibly important modification to the interpretation of the SSS concept: The undrained SSS is the minimum shear strength of a nonliquefied soil and the maximum shear strength of a liquefied soil. The minimum shear strength of a liquefied soil would occur where the normal effective stresses are zero, and would depend on the void ratio of the liquefied material, as well as on the velocity at the flow boundary.

Thus, in the writer's opinion, the corrected undrained SSS determined from field and laboratory tests as presented in this paper, applied to an analysis of the section shown in Fig. 3(*a*), would be an upper-bound analysis of the liquefied slope. A lower-bound analysis would result from assuming that the liquefied zone has zero strength. Other lower-bound cross sections could also be considered, but with respect to future use of the SSS concept for unfailed dams, a failed cross section would be unknown.

Discussion by John L. Vrymoed,[7] Member, ASCE

The writer is of the opinion that the undrained steady-state strength approach is fundamentally correct. In the case of Lower San Fernando Dam, however, the authors emphasize the shell and neglect the core's influence on the slide. This is a result of the authors' interpretation of the slide's failure mechanism.

The failure mechanism shown in Figs. 1 and 3 fails to explain the location of the core materials in the slide mass. Fig. 1 shows a cross section of station 12+80 where the core displaced about 100 ft upstream. This cross section, however, is outside the slide's main body. The main slide extended from the left abutment contact (station 0+00) to about station 12+00. From this station to the right abutment, the dam's cross-sectional height steadily decreases as does the extent of the slide.

The section shown in Fig. 1 does not represent the dam's maximum section. Sections taken at stations 9+00 and 5+00 are representative of the maximum section and the main body of the slide. These two sections show the core to be at the base of the slide and displaced 200 and 250 feet (Seed et al. 1973).

The authors postulate that cracking occurred at a location near the crest's

[7]Sr. Engr., Div. of Safety of Dams, California Dept. of Water Resour., 2200 X St., Sacramento, CA 95816.

upstream hinge point as shown in Fig. 3(*a*). This appears to be supported by Fig. 1, which shows that particular location of the dam to have been broken up into blocks numbered 4 and 5. This is not supported, however, when cross sections of the main slide are examined. These clearly indicate that a portion of the downstream slope, the entire crest and a significant portion of the upstream slope moved as one single unit without having been broken up (Seed et al. 1973).

The validity of the authors' failure mechanism is further doubted since it is indicated that the core materials did not experience a pore pressure increase during shaking (Seed et al. 1973; Seed 1979). This would mean that the core materials, still having their full effective strength, dramatically sheared, pushing up and flowing 200+ ft underneath the slide materials—40 s after the earthquake. If the core and shell did indeed experience respective pore pressure increases of zero% and 100%, the slide would have taken place without involving a majority of the core.

The analysis referred to by the authors that concludes that the slide occurred 40 s after shaking had stopped could not be located by the writer. Seed (1979) references the analysis to "G. Murray (1976) Private Communication." The writer is familiar with the comparison made with the seismoscope record and the aftershocks recorded on other accelerometers. How these aftershocks were so identified on the record and if the instrument's location (station 13+25) outside of the main slide was accounted for is unknown. It appears that this analysis is the sole basis for concluding that inertia forces did not play a role in the slide.

The writer finds it difficult to conceive how inertia forces could not have played a role since the base motions were estimated to have had a peak acceleration of 0.55g, and possibly higher (Scott 1973), and since the peak accelerations resulting in a safety factor of unity against sliding were estimated to range between 0.22g and 0.34g (Seed et al. 1973). These latter values were based on shear strengths developed at large strain levels. Based on lower and more appropriate strain levels, the writer has estimated that sliding was initiated by accelerations as low as 0.05g, It would therefore appear that inertia forces triggered the steady-state condition.

The authors' contention that the crest experienced 0.5g is misleading. Duke et al. (1972) report this value as the crest seismoscope's spectral acceleration. Since the seismoscope's 3/4-s period is near the dam's period, the spectral acceleration of 0.5g is probably the result of significant amplification of the crest motions by the seismoscope. Most likely, the actual crest accelerations were significantly less than 0.5g and, as the seismoscope record shows, were zero during most of the shaking.

An important component of the authors' failure mechanism is the postulated zone of liquefaction/high residual pore pressure in the upstream shell. As the authors indicate, the conditions in the downstream shell are the same as those in the upstream shell. High pore pressures should therefore also have developed in the downstream shell as predicted by Seed et al. (1973).

Piezometers 13, 64I, and 65O, located in the shell immediately downstream of the core, were measured 6 hours after the earthquake. The readings showed a maximum increase in head of 2 ft, an increase of 1%–2%. Other piezometers located further downstream showed similar negligible increases. It is doubtful that high pore pressures developed and that these dissipated in such a short amount of time. This would be contrary to the piezometer readings taken in Upper San Fernando Dam, which showed the pressures to linger for weeks and slowly decay after the earthquake.

Based on this, the writer doubts the authors' contention that a zone of high pore pressure developed in the upstream shell absent shearing during the earthquake. Any increase in pore pressure then must have been due to cyclic shear stresses. These were determined to be similar in both the downstream and upstream shells (Seed 1973). If cyclic shear stresses did not cause a pore pressure increase in the downstream shell then neither would they have caused an increase in the upstream shell.

Hence, the failure mechanism postulated by the authors was found to be a highly improbable sequence of events contradicted by field evidence. What is more probable is that shearing occurred simultaneously through the core and the upstream shell due to inertia forces. As a consequence, the large input motions were attenuated at crest. Evidence of increased pore pressures was as a result of undrained shearing. Shearing was minimized in the downstream direction by the buttressing effect of the berm. Hence, the corresponding negligible increase in pore pressure in this part of the dam.

Typically, hydraulic fill materials decrease in relative compaction toward the center of the dam. When the authors focus their attention on materials sampled downstream of the core, it is not surprising that "conservative to very conservative" strengths had to be used to predict the slide.

Sampling of the entire depth of the core over 2-ft intervals at station 4+00 was done in 1933 (LADWP 1933). The average dry density and moisture content based on 43 samples were 86.5 pcf and 37.3%. Most of the values of specific gravity and percent passing the number 200 sieve ranged between 2.70 and 2.79 and 95 and 99, respectively. Percentages of relative compaction based on Std. AASHTO (12,000 ft-lbs/ft3) ranged between 77 and 89 with an average of 83.

From 1933 up to the time of the slide, the vertical settlement of the crest was 0.8 ft. The densities may therefore have been slightly greater prior to the slide than those measured in 1933. How that change in density was distributed as a function of depth is unknown. Nevertheless, the dry densities of the core are significantly lower than those of the shell shown in Fig. 8.

Based on the 1933 findings, an average void ratio of 1.0 was computed. This value is far greater than any value of void ratio shown in the authors' figures. Choosing to place an emphasis on the core may therefore have a significant effect on the authors' analysis and may change their conclusion. Whether the authors choose to do this hinges on their choice of the slide's failure mechanism.

ACKNOWLEDGMENT

The writer thanks the Department of Water and Power, City of Los Angeles for providing access to the department's files. The department's cooperation and the assistance provided by Phil Lahr in an effort to the determine the slide's failure mechanism is sincerely appreciated.

The views expressed are solely the writer's and do not necessarily represent the views of the California Department of Water Resources.

APPENDIX. REFERENCES

Duke, C. M., Johnsen, K. E., Larson, L. E., and Engman, D. C. (1972). "Effects of site classification and distance on instrumental indices in the San Fernando Earthquake." *UCLA-ENG-7247*, Univ. of California, Los Angeles, Calif.

Scott, R. F. (1973). "The calculation of horizontal accelerations from seismoscope records." *Bull. Seismological Society America*, 63(5).
"Soil Compaction Curves, Test Hole 261." (1933). *File No. AX-215-9A*, Los Angeles Department of Water and Power, Los Angeles, Calif.

Closure by Gonzalo Castro,[8] Raymond B. Seed,[9] and Thomas O. Keller,[10] Members, ASCE

The discussion by Vrymoed raises a number of important points, and the writers appreciate his contribution. He notes the important role that the core materials played in the failure mechanism, a fact that the writers agree with, even though the role of the core was not sufficiently explained in the paper. The writers agree that the cross section through the slide shown in Fig. 1 of the paper is not within the area in which the largest slide movements developed. However, the cross section in Fig. 1 was shown because it corresponds to the location where a large trench was excavated (Seed et al. 1973) and thus is the section for which the most information is available about the condition of the sliding mass after the failure. The failure surface shown in Fig. 3 of the paper does not pass through the core, and thus it may lead the reader to conclude that the core played no role in the failure. Actually, the most critical failure surface as determined from stability analysis and from field observations after the failure does pass through the core, for example, see Castro et al. (1989). Data on the undrained strength of the core is presented in Seed et al. (1973) and Castro et al. (1989). An analysis of the dynamics of the failure in Davis et al. (1988) assumed that about 35% of the initial failure surface was within the core. In addition, once the failure was initiated and large strains were induced in the core, its strength was reduced to its steady-state strength, which was determined to be equal to about one-third to one-fourth of its peak value (Castro et al. 1989). Therefore, the large displacements of the slide mass that occurred were influenced strongly by the reduction of strength with strain in the core.

Vrymoed questions a statement in the paper concerning the role of inertia forces in the failure. The statement in the paper indicates that once the failure was initiated, the movements were not controlled by the seismic forces but only by the soil strengths along the failure surface, the weights of the masses involved, and the inertia associated with acceleration and deceleration of the masses as they started to move and as they stopped. Obviously, the seismic shaking was the triggering mechanism that initiated the failure. The writers' opinion that the very large slide movements that subsequently developed were controlled by the static forces and not by the seismic shaking is based not only on the delay between the end of the earthquake shaking and the beginning of the large movements, but also on the large unidirectional movements that constituted the slide as compared to the oscillating type of motion the seismic forces would generate.

Several of the points made by Vrymoed relate to the role of pore pressure increases within the dam. He mentions pore pressure increases of 2 ft or less measured about 6 hours after the failure in three observation wells, designated 64I, 64O, and 13, located in the downstream section of the dam. The tips of two of these observation wells (64I and 64O) are at about

[8]Princ., GEI Consultants, Inc., Winchester, MA 01890.
[9]Prof., Dept. of Civ. Engrg., Univ. of California, Berkeley, CA 94720.
[10]Sr. Project Mgr., GEI Consultants, Inc., Winchester, MA.

elevations 1,050 and 1,053, respectively, which are about 8 ft deeper than the pre-earthquake water levels. On the other hand, the critical zone of the hydraulic fill in typically between about elevations 1,010–1,040, i.e., about 10–40 ft below the tips of the observation wells. The tip of observation well 13 is at elevation 1,085, which is about 50 ft above the zone of interest. Thus, the readings in the three observation wells mentioned by Vrymoed do not reflect the pore pressure increases in the critical zone of the hydraulic fill. The writers' review of all available observation well data, as well as interviews with investigators involved in the original postslide investigations, indicated that no information was generated on the pore pressure increases that may have occurred in the soils that were critical to the failure, namely, the lower part of the hydraulic fill and the core material. In addition, heterogeneity of the fill material (with regard to permeability) resulting from interlayering of finer and coarser lenses and strata may have resulted in significant dissipation of pore pressures in the time that elapsed prior to examination of these wells.

Vrymoed comments on the pore pressure increases that may have occurred in the critical soils in both the upstream and downstream sections of the dam. He appears to imply that the triggering of the liquefaction failure is associated with the development of 100% pore pressure. This is not the case. The upstream liquefaction (stability) failure took place because the accumulated strains in the hydraulic fill induced by the earthquake were sufficient to cause its strength to drop to S_{us}. Subsequent large movements also caused the strength of the core to drop to its S_{us}, which further decreased the stability and caused the movements to become even larger. The estimated undrained steady-state strength S_{us} of the hydraulic fill is equal to about 15% of the corresponding drained strength of the soil, and thus during the failure the maximum pore pressure increase in this soil would have been equal to about 85% of the initial effective stress. At the time the failure was triggered, the pore pressure ratio would have been in the range of approximately 40%–50% (Vasquez-Herrera and Dobry 1989). Thus, the failure mechanism does not require the pore pressure ratio to reach 100% in the critical zone of the hydraulic fill in the upstream section of the dam. Additional work confirming this conclusion is presented in Boulanger et al. (1991).

The downstream section of the dam was somewhat more stable than the upstream section due to differences in geometry, including the presence of a downstream beam. A stability failure of the downstream section was not possible even if the strength of the downstream hydraulic fill was reduced to S_{us} (Castro et al. 1989). It is not known what values of pore pressure developed in the critical zone of the hydraulic fill in the downstream section. However, regardless of the value of these pore pressures, a downstream stability failure resulting in large downstream displacements could not have occurred.

Day and Hovland present important comments in their discussions relative to the mechanism of the movements of the various blocks of the upstream section of the dam as shown in Fig. 1(a) of the paper. The reader is also referred to similar depictions of the movements for other sections of the dam as shown in Seed et al. (1973). The discussers note that the blocks near the toe moved farther than the blocks closer to the center of the dam. One possible explanation for this observation is that as the soil mass moved into the reservoir, it may have trapped water under the mass, and therefore, the shear resistance along its base may have been lower than the value of

S_{us} of the hydraulic fill itself, thus causing the soils to find lesser resistance and accelerate as they moved into the reservoir. A second possible explanation is slumping of the underlying softened soils, resulting in localized spreading between the relatively intact blocks.

Day presents a possible explanation for the time delay between the end of earthquake shaking and the beginning of the slide, involving progressive failure. The writers believe that progressive failure most likely was a contributing factor, and thus that Day's explanation is reasonable. Other possible reasons that may have contributed to the delay in the failure are:

1. Accumulated shear strains in some zones of the critical layer may not have been quite enough to reduce its strength to a level equal to the driving shear stress, and additional creep under the static driving stress was needed to lower its strength sufficiently to cause a flow slide. Creep was observed in triggering tests as reported in Castro et al. (1989).
2. The dense sandy starter dike at the toe of the upstream slope was dilative, and during undrained shear there was a reduction in pore pressures that caused the strength of the toe dike to be significantly greater than its drained strength. The failure occurred when the strength was gradually reduced to a lower value nearer to its drained value as water from the reservoir flowed into the dilating soil. This mechanism of the delay in the slide was proposed by Seed (1979).

Both Day and Hovland comment on the comparison between the estimated strength of the hydraulic fill backfigured from the slide and the strength estimated for laboratory tests, as presented in Table 2 of the paper. There are three key issues that affect the comparison.

First, there are substantial uncertainties in backfiguring the strength from the slide as some of the responses to Day and Hovland's discussions illustrate. The consensus range of 400–500 psf does not fully reflect the range of the uncertainty. The full range of values obtained by the authors is larger than this consensus range. The consensus range represents values that all authors considered possible even though the range of probable values are different for each author. Castro and Keller agree with the estimated value in Davis et al. (1988) of 540 psf, which considers explicitly the acceleration and deceleration of the sliding mass. The value of 540 psf is a best estimate, and the uncertainty is about ± 100 psf. Both Raymond B. Seed's and H. Bolton Seed's views are reflected in Seed et al. (1989), which concludes that the S_{us} value was in the range of 300–500 psf, with a best estimate of 400 psf. The lower end of this range corresponds to the value of S_{us} required for stability of the final configuration of the sliding mass, plus a slight increase (~30–50 psf) for momentum effects. Sources of uncertainty in backfiguring the strength of the hydraulic fill from the slide are many including: (a) Resistance offered to the sliding mass as it entered the reservoir; (b) strength mobilized in soils other than the hydraulic fill, i.e., core, rolled fill, ground shale hydraulic fill, soil in the starter dikes; (c) the sequence of movements of the various blocks within the slide including progressive failure, and (d) the actual duration of the movements which influences any computation that considers the dynamics of the failure (including deceleration of the slide mass, or momentum effects).

Second, the values of the strengths estimated from the laboratory tests are a function of the correction that needs to be applied to the test results obtained on 1985 samples from the downstream shell to represent 1971 pre-earthquake conditions of soils in the upstream shell. Two procedures to perform the correction are referred to as methods A and B in Table 2 of the paper. For detailed explanations of the two methods, the reader is referred to Seed et al. (1989) and Castro et al. (1989).

The third issue is the selection of a representative value from the strengths estimated from the laboratory test results. The sliding zone within the hydraulic fill was large, generally 10–30 ft thick, and it was horizontally stratified as a result of the hydraulic method of deposition. Thus, the average mobilized strength during the failure would be controlled primarily by the weaker materials and not by the average strength. Inclusions of stronger materials will simply float within the weaker material as the slide takes place. Accordingly, it is difficult to accurately determine the appropriate value for comparing with the backfigured strength. The average minus one-half of the standard deviation (or 33 percentile) strengths listed in Table 2 (490 and 650 psf for void ratio correction methods A and B, respectively) are considered a reasonable approximation of the mobilized strength.

How well the results of the laboratory tests agree with the backfigured strength depends on one's opinion concerning the key issues discussed here. Based on the range of opinions presented above, the ratio of the representative value of the laboratory test results (33 percentile) to backfigured strength range from about 0.9 (490/540) to 1.6 (650/400). The first ratio represents the lowest laboratory tests 33 percentile value (490 psf) and the highest ratio of the two best estimates for the backfigured strength (540 psf). The second ratio represents the highest laboratory 33 percentile value (650 psf) and the lowest best estimate of the backfigured strength (400 psf). The writers consider the level of agreement represented by the ratio of 0.9 to 1.6 to be reasonable given the many uncertainties involved in both the correction of the laboratory test results to pre-earthquake conditions and in backfiguring of the actual field strength from the failure. Furthermore, it is important to note that under all interpretations the test results indicate that the upstream section of the dam was susceptible to at least some significant level of sliding as a result of an earthquake, and that the downstream section was not, both results being in agreement with the observed behavior.

APPENDIX. REFERENCES

Boulanger, R. W., Seed, R. B., Chan, C. K., Seed, H. B., and Sousa, J. (1991). "Liquefaction behavior of saturated sands under uni-directional and bi-directional monotonic and cyclic simple shear loading." *Report No. UCB/GT/91-08*, Univ. of California, Berkeley, Calif.

Castro, G., Keller, T. O., and Boynton, S. S. (1989). "Re-evaluation of the Lower San Fernando Dam." *Report 1, Contract Report GL-89-2*, U.S. Army Waterways Experiment Station, Vicksburg, Miss.

Seed, H. B., Seed, R. B., Harder, L. F., and Long, H.-L. (1989). "Reevaluation of the Lower San Fernando Dam." *Report 2, Contract Report GL-89-2*, U.S. Waterways Experiment Station, Vicksburg, Miss.

Vasquez-Herrera, A. and Dobry, R. (1989). "Reevaluation of the Lower San Fernando Dam." *Report 3, Contract Report GL-89-2*, U.S. Army Waterways Experiment Station, Vicksburg, Miss.